Tension Tests on Cast-in-Place Inserts: The Influence of Reinforcement and Prestress

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Tension tests were conducted on four different types of cast-in-place inserts, including both wire-formed and bolt-type inserts, to investigate their behavior under conditions present in prestressed concrete bridge girders. The effects of parameters such as the level of axial compression, reinforcement details, limited edge distance, and interaction of double inserts on the behavior of the inserts were studied. Concrete breakout failures occurred in the majority of the specimens. Concrete breakout capacities measured for the unreinforced specimens corresponded closely to the breakout capacities predicted by ACI 318-05 equations for cracked concrete. Specimens with loop inserts exhibited stable and ductile behavior as a result of extensive plastic deformation of the insert. The other three types of inserts exhibited larger tensile load capacities than the loop inserts in the reinforced specimen. However, they did not have as much ductility. The behavior of the loop inserts was not significantly influenced by the test variables studied; the behavior of specimens with the other three types of inserts varied depending mainly on the reinforcement details.

C ast-in-place, wire-formed inserts have been widely used in precast, prestressed concrete members to facilitate connections between elements. One particular example of their application is connecting cast-in-place concrete diaphragms and floor beams to precast concrete girders in bridges. As the only connection between the diaphragm and the girders, behavior of the bridge system under vertical and lateral loads is dependent on the behavior of these inserts.

Despite their important role in transferring loads between
components and providing integrity to the bridge system, there is limited information available on the behavior of wire-formed inserts, especially when they are placed in concrete members containing mild steel and prestressing reinforcement. As part of a PCI-sponsored research program, a recent study has been conducted by Anderson and Meinheit on the pryout capacity of cast-in-place headed studs. Even though the study involved tests on stud groups, as well as an extensive review of test data reported in the literature, the scope was limited to shear push-off tests and pryout-type failures of headed-stud anchors. Therefore, there is still a need for information on the behavior of wire-formed inserts under tensile loading.

This paper presents the results of tensile tests performed on various types of wire-formed and bolt cast-in-place inserts to investigate their behavior under circumstances present in prestressed concrete bridge girders. Load and deformation capacities obtained from the tension tests were analyzed and compared to identify the effects of the test variables. Concrete breakout capacities measured for some of the speci-

Table 1. Summary of Test Variables in Research Program

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Insert Type</th>
<th>Insert Location</th>
<th>No. of Inserts</th>
<th>Axial Compression, ksi</th>
<th>Longitudinal Reinforcement</th>
<th>Stirrup Spacing, in.</th>
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<td>Thin-slab</td>
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<td>Unreinforced</td>
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<td>1</td>
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</table>
mens were also compared with the capacities predicted by the equations in ACI 318-05, Appendix D.

SPECIMEN DESCRIPTION

The experimental investigation involved testing 32 tension specimens. Table 1 lists the test specimens and associated variables, including the type, location, and number of inserts; the presence of axial compression to represent the effect of prestress; and the presence of transverse and longitudinal reinforcement. Each specimen is identified by a combination of letters and numbers, as represented in Fig. 1. Figure 2 shows specimen dimensions and insert and reinforcement locations.

Types of Inserts

The four types of inserts tested in this study were loop, thin-slab, double-leg, and bolt. Geometric details of the inserts are given in Fig. 3. The first three types of inserts are commercially available, while the bolt insert was fabricated specifically for the pull-out tests and consisted of a 3/4-in.-diameter (20 mm) ASTM A 490 bolt, a coupler, and a washer.

The loop insert represents a typical insert used in prestressed concrete bridge girders in the state of Minnesota. The other three types of inserts included in the study represent potential alternatives. A selection criterion was that all inserts had to be able to be placed between tensioned prestressing strands that were spaced at 2 in. (51 mm) on center in the concrete girders.

Location and Number of Inserts

As mentioned, a particular example for the use of wire-formed inserts is to connect cast-in-place concrete floor beams to precast concrete girders. Figure 4 shows a typical connection detail used in the construction of prestressed concrete through-girder pedestrian bridges in Minnesota. At each floor beam location, there are six wire-formed inserts,
two in the flange and four in the web of the girder.

The testing program included inserts placed at the center of the specimen (center-insert specimens) and inserts placed closer to an edge of the specimen (edge-insert specimens). The center-insert specimens represent the case when inserts are placed in the web of the girder, and the edge-insert specimens represent when the inserts are in the bottom flange of the girder. To investigate the interaction among the failure areas of individual inserts, some of the specimens contained two inserts.

In the center-insert specimens shown in Fig. 2, the inserts were placed at the center of a 20-in.-wide (508 mm) concrete block. The specimen width provided 10 in. (254 mm) of distance on either side of the insert to reduce edge effects. The minimum edge distance of 10 in. (254 mm) used in center-insert specimens was equal to 2.6 to 3.2 times the embedment depth for the four types of inserts used.

The edge-insert specimens had a nominal width of 13.75 in. (349 mm), with the inserts placed at 3.75 in. (95 mm) from the edge of the specimen. The 3.75 in. (95 mm) side distance was used in the edge-insert specimens to approximate the side distance of the inserts embedded in the bottom flange of the prestressed concrete girders used in through-girder bridge construction.

In the double-insert specimens, the inserts were spaced at either 6 in. or 4 in. (152 mm and 102 mm) on center depending on whether it was a center- or edge-insert specimen, respectively. In the double-center-insert specimens, 6 in. (152 mm) insert spacing was used because this is the spacing between inserts placed in the web of prestressed concrete girders used in through-girder pedestrian bridges. The double-edge-insert specimens included 4-in.-spaced (102 mm) inserts because the inserts placed in the bottom flange of girders typically have 4 in. (102 mm) spacing.

Fig. 4. Relationship between prototype and test specimens.
Presence of Axial Compression

One of the objectives of the experiments was to evaluate the behavior of inserts under circumstances present in prestressed concrete girders used in through-girder pedestrian bridges, for which a typical design results in a net compressive stress in the bottom flange. No studies could be found in the literature regarding the effect, if any, of axial compression on the behavior of embedded inserts.

In this study, compressive stress due to prestressing was simulated by a hydraulic testing machine that applied an external axial compressive load to the specimens. As shown in Table 1, the majority of the specimens were tested under 1 ksi (6.9 MPa) of axial compressive stress. There were also some specimens tested under 2 ksi (13.8 MPa) of compressive stress, as well as others subject to no axial compression. The compressive stress at insert locations in a typical prestressed concrete girder in a through-girder pedestrian bridge under the self-weight of the bridge was determined to be between 1.2 ksi and 2.5 ksi (8.3 MPa and 17.2 MPa). The compressive stress levels used in the tests were representative of those observed in the bridges under service.

Presence of Reinforcement

In an attempt to simulate the restraining effect that the prestressing strands and transverse reinforcement might have on the behavior of inserts placed in bridge girders, both longitudinal and transverse reinforcement were incorporated into some of the specimens.

The dowel action provided by the prestressing strands in a prestressed concrete girder was simulated through ½-in.-diameter (12.7 mm) steel threaded rods that were wrapped in tape and greased to prevent bonding with the surrounding concrete. This enabled the rods to remain unstrained when a prestressing force was imposed on the test specimens through the application of the external compressive load, as described previously. Washers and nuts were placed at the ends of the threaded rods so that the dowel action of the rods could be obtained when the tension load on the inserts was transferred to the rods. Figure 5 shows the placement of inserts and reinforcement in the casting forms for the edge-insert specimens.

In the edge-insert specimens, the longitudinal rods were located to simulate the location of the prestressing strands from the top surface of the bottom flange of a prestressed concrete I-girder. In the edge-insert specimens with thin-slab inserts, the position of one of the longitudinal rods was modified slightly in order to position the inserts between the rods.

Number 4 (13M) stirrups were used as transverse reinforcement in the specimens. The majority of the specimens contained stirrups spaced at 6 in. (152 mm), the typical stirrup spacing used in the vicinity of inserts in through-girder pedestrian bridges in Minnesota. To investigate the influence of stirrup use and spacing, some of the specimens were tested with stirrups at a 4 in. (102 mm) spacing, while others were tested without any stirrups.

In the remainder of this paper, the term reinforced specimens refers to specimens that contain both longitudinal rods and stirrups, while the term unreinforced specimens refers to specimens that contain neither longitudinal rods nor stirrups.

CONCRETE PROPERTIES

The specimens were fabricated in two separate concrete placements at the Structural Engineering Laboratory of the Department of Civil Engineering at the University of Minnesota using commercially available ready-mixed concrete. The designation used for each beam indicates whether the specimen was fabricated in the first or second cast (1 denotes the first concrete placement, and 2 denotes the second).

The concrete strengths were averaged from testing three 4 in. x 8 in. (102 mm x 203 mm) cylinders. The 28-day compressive strengths $f'_c$ of the two concrete batches were 9200 psi (63 MPa) for the first cast and 8600 psi (59 MPa) for the second cast. The 28-day split cylinder strengths $f'_t$ were 660 psi and 580 psi (4.6 MPa and 4.0 MPa) for the first and second casts, respectively. The concrete strengths in both casts were fairly representative of those used in typical prestressed concrete bridge girders.

At the time of load tests, the ages of the first-cast specimens were between 139 days and 148 days, while the second-cast specimens were tested at ages between 34 days and 80 days. Figure 6 shows the change in concrete compressive strength with time for concrete cylinders from both casts. Because the second-cast specimens were subjected to load tests at early ages, the concrete strengths were monitored more frequently for the second cast. Measured splitting strength values are

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**Fig. 5.** Placement of insert and reinforcement in edge-insert specimens.
also tabulated in Fig. 6.

The concrete compressive strengths of the specimens at the time of load testing were between 10,600 psi and 10,800 psi (73 MPa and 74 MPa) (approximately 2% difference) for the first-cast specimens and between 9200 psi and 9500 psi (63 MPa and 65 MPa) (approximately 3% difference) for the second-cast specimens. As a result of this small variation in concrete strengths of specimens belonging to the same concrete placement, a direct comparison of the behavior among the same-cast specimens was possible.

TEST SETUP AND INSTRUMENTATION

The test setup shown in Fig. 7 was different from that prescribed by ASTM E 488 for tensile anchor tests. In the conventional (ASTM) setup, the load is applied to the anchor by a hydraulic jack that reacts against the specimen through a circular steel reaction ring or a reaction assembly made up of steel sections. In this type of setup, compression struts between the anchor and the reaction points on the surface of the specimen might influence the anchor behavior. To eliminate such interaction, the reaction points should be located outside the expected concrete failure area.

The setup used in this study was a modified form of the setup recommended by ASTM E 488 for cyclic tension testing of anchors. In the configuration used in this study, the specimens were placed underneath a 600-kip-capacity (2700 kN) MTS Systems Corp. universal testing machine. As shown in Fig. 7, the axial compression force was applied to the specimen by the testing machine through steel I-sections placed above and below the specimen. Tension load from a horizontally positioned 77-kip-capacity (340 kN) hydraulic actuator was transferred to the insert through a ¾-in.-diameter (19 mm), high-strength steel threaded rod. For the double-insert specimens, the tension load was first transferred from the hydraulic actuator to a steel plate through a steel threaded rod and then transferred to the inserts through two steel threaded rods.

The specimen was supported along the back of its top and bottom surfaces by three ¾-in.-diameter (22 mm), high-strength steel threaded rods, which were embedded in the concrete and reacted against the steel I-sections located above and below the specimens. The ¾-in.-diameter (22 mm) threaded rods used to support the specimens were placed behind the inserts; therefore, they did not influence the behavior of the inserts. This setup also eliminated any possible interaction between the applied load and the concrete breakout area.

Figure 7 shows a detail of the connection used at the top and bottom of the specimens. The adapter plates shown in the figure transferred the compression force applied by the testing machine directly to the concrete while the longitudinal rods, representing the dowel effects of the prestressing strands, remained stress-free. As mentioned, bond between the longitudinal rods and the concrete was prevented to keep the rods free of stress. To prevent slipping of the rods inside the concrete out during the load tests, steel nuts were placed at both ends of the longitudinal rods.

Linear variable displacement transducers (LVDTs) were used to monitor the insert displacements as well as the displacement of the front and back faces of the concrete blocks. The amount of deformation of the insert was determined by...
measuring the movement of a point located on the threaded rod, approximately 1 in. (25 mm) away from the front face of the specimen. The threaded rod transferred the tensile load from the hydraulic actuator to the inserts. Figure 7 indicates the location of the point from which the insert displacements were measured. Displacement of this point on the threaded rod was measured relative to a fixed point on the laboratory floor. Displacements recorded by the LVDTs placed at the back face of the specimen were used to correct the insert displacements for tilting of the specimen.

Strain gauges were placed on the surfaces of two specimens (Specimens 1-UC-T-0-N and 1-RC-T-1-6) prior to load tests in order to determine whether the distribution of compressive strain in the cross section of the specimens was uniform. Two gauges were placed on two sides of the specimen. Strains measured from these gauges indicated that a uniform compression of the concrete blocks was achieved during testing.

Four of the specimens (Specimens 2-RC-L-1-6, 2-RC-T-1-6, 2-RC-D-1-6, and 2-RC-B-1-6) also had strain gauges placed on the longitudinal rods and stirrups near the insert location. As indicated in Fig. 8, one strain gauge was placed on the top surface of each rod and stirrup. Data from these gauges were used to determine the extent to which the longitudinal rods and stirrups were engaged in the load-carrying mechanism for each type of insert.

The specimens were also whitewashed with a mixture of lime and water to ease visual detection of cracks during load testing.

Fig. 8. Strain gauges placed on longitudinal rods and stirrups.

Fig. 9. Typical deformation patterns for the inserts in reinforced specimens.

Table 2. Test Results

<table>
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<th>Specimen Designation</th>
<th>Failure Type</th>
<th>Load Capacity, kip</th>
<th>Testing Date</th>
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<td>10/28/04</td>
</tr>
<tr>
<td>2-RE-L-1-4</td>
<td>Steel and weld</td>
<td>14.1</td>
<td>10/29/04</td>
</tr>
<tr>
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<td>Concrete</td>
<td>14.4</td>
<td>11/9/04</td>
</tr>
<tr>
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<td>Concrete</td>
<td>13.2</td>
<td>11/9/04</td>
</tr>
<tr>
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<td>11/12/04</td>
</tr>
<tr>
<td>2-RED-L-1-6</td>
<td>Concrete</td>
<td>28.6</td>
<td>11/16/04</td>
</tr>
</tbody>
</table>

*Fabrication date of first-cast specimens was May 4, 2004, and second-cast specimens was September 2, 2004. Note: 1 kip = 4.448 kN.

**TESTING OF SPECIMENS**

The testing sequence included two loading phases. In the first phase, a compressive axial load was applied to the test specimen with the 600-kip-capacity (2700 kN) universal test-
ing machine operated in force-controlled mode, except for the specimens to be tested with no axial compression. The vertical load was applied over a period of five minutes. After reaching the full load, the loading was paused and the amount of compressive axial load on the specimen was held constant during the second loading phase.

In the second loading phase, a tension load was applied to the insert using the horizontally positioned 77-kip-capacity (340 kN) hydraulic actuator in displacement-controlled mode. A typical loading rate of 0.0833 in./min (2.116 mm/min) was used during this phase of loading and the specimen was loaded until failure. During the tests, careful visual inspection of the specimens was conducted to monitor development of surface cracking.

### FAILURE MODES

Table 2 identifies the failure mode for each specimen as a concrete-, steel-, or weld-type failure. In all but the loop-type insert tests, the specimens experienced concrete failures. Steel failures were observed with many of the loop inserts. A common feature of all modes of failure in the majority of the specimens was the formation of radial cracks, which began at the insert location and propagated outward on the front face of the specimens.

#### Steel and Weld Failures

As indicated in Table 2, steel failure occurred in six of the eleven specimens with loop inserts: Specimens 1-UC-L-0-N, 1-RC-L-1-6, 2-RC-L-2-6, 2-RE-L-2-6, and 2-RE-L-1-4. In five of these specimens, two legs of the loop insert ruptured at the corner locations as shown in Fig. 9. In the sixth of these specimens, Specimen 2-RE-L-1-4, a combination of steel and weld failure occurred. In Specimen 2-RCD-L-1-6, one of the two loop inserts experienced weld failure at the location where the legs connected to the coupler part of the insert.

Figure 10 shows typical concrete breakout patterns observed in specimens exhibiting steel failure. In these specimens, there was relatively minor damage to the concrete with little or no breakout of the concrete cover.

In the remaining four specimens with loop inserts (Speci-
a regular concrete cone breakout or concrete breakout with radial cracking. Figure 11 shows examples of these two cases.

In the case of reinforced, center-insert specimens, concrete breakout occurred with irregularly shaped failure areas, as shown in Fig. 12. In Specimen 1-RC-B-1-6, splitting of the entire front face of the specimen occurred. In general, concrete breakout depths observed in these specimens were smaller than those observed in unreinforced, center-insert specimens. The inserts in these specimens experienced more inelastic deformation than the inserts in unreinforced, center-insert specimens due to the restraining effects of longitudinal rods. As a result, these specimens exhibited more ductile behavior with larger force capacity than the unreinforced, center-insert specimens.

Figure 13 illustrates the two distinct concrete breakout patterns observed in the edge-insert specimens. The limited edge distances used in these specimens resulted in concrete edge breakout failures. Some of the specimens in this group exhibited shallow breakout cones, while others exhibited more typical breakout cones (Fig. 13).

Failure Cone Inclination Angles

Figure 14 gives the concrete breakout cone profiles observed in the unreinforced specimens. The plotted breakout cone profiles are shown for a vertical section passing through the center of the inserts. The figure includes the cone profiles for the four types of inserts tested with two specimens for each insert type.

Specimens with double-leg and bolt inserts exhibited consistent breakout cone profiles. The shallow breakout cone observed in Specimen 1-UC-L-0-N was due to a steel failure that occurred in this specimen. The unsymmetrical concrete breakout cone of Specimen 2-UC-T-0-N suggests that, in addition to tensile load, some amount of downward shear loading might have been exerted on the insert.

Failure cone inclination angles determined using the profiles shown in Fig. 14 are tabulated in Table 3. The table shows the inclination angles from a horizontal plane determined for the two sides of the profiles and their average. By excluding Specimen 1-UC-L-0-N, which had a steel failure, the average inclination angles observed in the specimens ranged from 29 to 45 degrees. These inclination angles were consistent with the 30- to 45-degree angles explicitly or implicitly recommended in different resources to predict the capacity of headed anchors in concrete under tensile loads.\textsuperscript{24, 8}

INTERNAL STEEL STRAINS

As mentioned, longitudinal rods and stirrups in the second-cast, reinforced center-insert specimens were instrumented with strain gauges during fabrication. Figure 15 plots the strains measured by these gauges during the load tests. For
Fig. 14. Concrete breakout cone patterns observed in unreinforced specimens. Note: 1 in. = 25.4 mm.

each specimen, strains from one strain gauge placed on a stirrup and one placed on a longitudinal rod in the vicinity of inserts were included in the plots. The strain gauges indicated tensile strain from axial tension and bending of the stirrups and the longitudinal rods.

The reason that the strain gauges in Specimen 2-RC-L-1-6 measured smaller strain values than those in the other three specimens is related to the load-carrying mechanism of the loop insert (Fig. 15). In Specimens 2-RC-T-1-6, 2-RC-D-1-6, and 2-RC-B-1-6, the forces generated by the inserts were resisted by the longitudinal rods, which were restrained by stirrups as they carried the applied load. In these specimens, part of the load applied to the insert was transferred to the longitudinal rods and the stirrups, causing large tensile strains in these elements at the strain gauge locations.

In Specimen 2-RC-L-1-6, alternatively, there was no direct transfer of load from the insert to the longitudinal rods or the stirrups because of the geometry of the loop insert (Fig. 15). This resulted in lesser measured steel strains in this specimen.

In all four specimens, load versus strain plots for stirrups showed a change in the initial slope at some values of applied load (Fig. 15). After the slope change, the stirrup strains increased at a higher rate. The increase in the rate of change of stirrup strains was due to the formation of cracks crossing the stirrups.

The fact that the initiation of cracking was always evident in the stirrup strain measurements and not in the strains measured on the longitudinal rods might be a coincidence, given that the first cracks always crossed the stirrups but not the longitudinal rods. Another reason that initiation of cracking wasn’t evident in the rod strains might be that they were debonded from the surrounding concrete, so the initiation of cracks in the concrete at the gauge locations did not cause nearly as much appreciable additional strain in the rods as in the stirrups.

The loads at crack initiation determined using the point of divergence in load versus steel strain plots are identified in Fig. 15. The load at crack initiation for Specimens 2-RC-L-1-6, 2-RC-T-1-6, 2-RC-D-1-6, and 2-RC-B-1-6 were 5.2 kip, 8.1 kip, 10.4 kip, and 13.9 kip (23.1 kN, 36.0 kN, 46.3 kN, and 61.8 kN), respectively. During load testing, radial cracks beginning at the insert location and propagating outward on the front face of specimens were visually detected at loads of 16 kip and 30 kip (71.2 kN and 133.4 kN) in Specimens 2-RC-T-1-6 and 2-RC-B-1-6, respectively. No radial cracking was visually detected in Specimens 2-RC-L-1-6 and 2-RC-D-1-6.

The cracking load levels determined from the internal steel strains were consistent with observations reported by Stone and Carino. They used the results from testing two large-scale, headed-insert specimens that were heavily in-

Table 3. Failure Cone Inclination Angles Observed in Unreinforced Specimens

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>Top Angle, degrees</th>
<th>Bottom Angle, degrees</th>
<th>Average Angle, degrees</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-UC-L-0-N*</td>
<td>21</td>
<td>16</td>
<td>19</td>
</tr>
<tr>
<td>2-UC-L-0-N</td>
<td>39</td>
<td>42</td>
<td>41</td>
</tr>
<tr>
<td>1-UC-T-0-N</td>
<td>33</td>
<td>34</td>
<td>34</td>
</tr>
<tr>
<td>2-UC-T-0-N</td>
<td>40</td>
<td>18</td>
<td>29</td>
</tr>
<tr>
<td>1-UC-D-0-N</td>
<td>26</td>
<td>31</td>
<td>29</td>
</tr>
<tr>
<td>2-UC-D-0-N</td>
<td>31</td>
<td>28</td>
<td>30</td>
</tr>
<tr>
<td>1-UC-B-0-N</td>
<td>47*</td>
<td>42</td>
<td>45</td>
</tr>
<tr>
<td>2-UC-B-0-N</td>
<td>31</td>
<td>44</td>
<td>38</td>
</tr>
</tbody>
</table>

*Steel failure.
instrumented with internal concrete strain gauges to determine the formation and propagation of internal cracking during the pull-out tests. Circumferential cracking was determined to begin near the bottom of the inserts at load levels that were 25% to 35% of the ultimate load. It was also reported that radial cracking began at lower load levels.

**LOAD-DEFLECTION BEHAVIOR**

The load-deflection responses of the specimens, obtained from the load tests, were evaluated with respect to several variables, such as insert type, insert location, and presence of reinforcement. The load values used in the load-deflection plots in this paper were the tensile loads applied to the inserts by the laterally positioned hydraulic actuator, and displacement values in the plots represent the horizontal movement of the insert as measured by the LVDTs located approximately 1 in. (25 mm) away from the front face of the specimen on the rod transferring the load from hydraulic actuator to the insert (Fig. 7). The insert displacements were corrected for any tilting or movement of specimens using the displacement measurements taken on the back side of the specimen.

**Unreinforced Specimens**

**Figure 16** gives the load-deflection responses of the unreinforced specimens tested with no axial compression. Pictures of the specimens and the inserts following the load tests are inset in the plots. There were a total of eight specimens in this group, and the plots are arranged with respect to the insert type. For each insert type, there were two repeat specimens, one of them from the first cast and the other from the second cast. Table 2 gives the value of the maximum load carried by each specimen.

As mentioned previously, there were differences in the measured concrete properties (compressive and split tensile strengths) between the two casts. The deviations observed between the response of the first cast and the second cast (or repeat) specimens were attributed to these differences in concrete strengths.

Splitting tensile strengths of the concrete used in the first placement were as much as 24% higher than the split tensile strengths of the second placement, while the differences between the measured concrete compressive strengths of specimens from the first and second casts were between 13% and 17%. For the three types of inserts that resulted in concrete failure (thin-slab, double-leg, and bolt), specimens from the first cast had larger load capacities than those from the second cast. The ratios of load capacities of the first cast and second cast specimens were 1.13, 1.39, and 1.21 for those containing thin-slab, double-leg, and bolt inserts, respectively.

As shown in **Fig. 16**, specimens with the loop insert exhibited ductile behavior as a result of substantial plastic deformation (necking at several locations on the loop). Indeed, failure of Specimen 1-UC-L-0-N was due to fracture of the insert legs, as shown in the inset to its plot. The maximum loads attained by the two loop-insert, unreinforced specimens
Fig. 16. Load-deflection curves for unreinforced specimens. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

Fig. 17. Load-deflection curves for reinforced center-insert specimens. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.
were very similar, even though some differences are observed between the load-deflection plots in early stages of loading.

Different from the loop inserts behavior, behaviors of the thin-slab, double-leg, and bolt inserts, as shown in Fig. 16, were brittle. These specimens failed shortly after reaching their maximum loads (due to concrete breakout failure) and before the inserts achieved substantial plastic deformation.

**Reinforced Center-Insert Specimens**

*Figure 17* gives the results of the reinforced specimens tested under 1 ksi (6.9 MPa) axial compressive stress. There were small differences between the load-deflection plots of the first- and second-cast specimens, even though the general behavior was the same.

Similar to the unreinforced specimens, the first-cast specimens had larger load capacities than the second-cast specimens for the three types of inserts that resulted in concrete failure (thin-slab, double-leg, and bolt inserts). The ratios of load capacities of the first-cast and second-cast specimens were 1.21, 1.12, and 1.03 for thin-slab insert, double-leg insert, and bolt insert, respectively. As mentioned, the differences between the measured load capacities of the first-cast and second-cast specimens were attributed to the difference in concrete strengths between these two series of specimens.

*Figure 18* compares the load-deflection curves of the reinforced and unreinforced specimens. For each insert type, results from two unreinforced specimens tested with no axial load and two reinforced specimens with an axial compressive stress of 1 ksi (6.9 MPa) are shown in the figure.

It should be noted that the concrete strengths measured at the time of load testing of the specimens differed by 2% among the first-cast specimens and by 3% among the second-cast specimens, as explained previously. As a result, the effect of concrete strength on the behavior of specimens from the same cast remained minimal.

The influence of reinforcement and axial compression is clearly evident in *Fig. 18* for specimens with thin-slab and bolt inserts. For these specimens, the presence of reinforcement and axial compression resulted in major increases in the load capacity of the specimens as well as improvements in ductility. As shown in *Fig. 18*, the improvement in ductility is more pronounced for the thin-slab-insert specimens, while the increase in load capacity is larger for the bolt-insert specimens. The average increase in load capacity was 54% and 123% for the thin-slab and bolt inserts, respectively.

The increases in load capacity and ductility were attributed to two things. First, the restraining effect provided by the longitudinal rods and stirrups caused the inserts to undergo large plastic deformations. The presence of longitudinal rods and stirrups also resulted in a larger area of concrete breakout, which increased the load capacity of the inserts. Second, the

![Fig. 18. Comparison of load-deflection curves for unreinforced and reinforced center-insert specimens. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.](image-url)
axial compression delayed the concrete cracking in the direction perpendicular to the axial compression, which may have increased the inserts' capacities.

The reason that the double-leg inserts were not influenced by the restraining effects of the longitudinal rods and stirrups as much as the thin-slab and bolt inserts was probably the relatively lesser flexural stiffness of the legs of the double-leg inserts.

Specimens containing loop inserts showed similar behavior to each other irrespective of whether there was reinforcement and axial compression. As mentioned, the behavior of the specimens with loop inserts was mainly controlled by the plastic deformation of the insert itself. As a result, including reinforcement and axial compression did not have a major effect on the behavior of the loop-insert specimens. As indicated in Fig. 18, three of the specimens with loop inserts exhibited steel failure, meaning fracture of the inserts. Even though the fourth specimen exhibited concrete failure, substantial plastic deformation of the insert, including pronounced necking of the legs, was observed prior to failure.

**Interaction of Multiple Inserts**—Figure 19 presents the effect of the interaction of multiple inserts for the loop and thin-slab specimens. Each plot includes results from one single-insert specimen and one double-insert specimen, both of which were reinforced specimens and tested under 1 ksi (6.9 MPa) axial compressive stress. Only the results from the specimens fabricated in the second cast are included in these plots. In this manner, the effect of any difference in concrete properties is minimized.

In the case of the loop insert, the double-insert specimen exhibited twice the load capacity of the single-insert specimen (Fig. 19). This result indicates that loop inserts spaced 6 in. (152 mm) apart do not interact with each other. In the double-insert specimen, premature failure of one of the inserts occurred due to weld fracture.

In contrast to the behavior of the loop inserts, the thin-slab inserts spaced at a 6 in. (152 mm) distance were observed to interact with each other (Fig. 19). The load capacity of the specimen with two thin-slab inserts was determined to be only 46% larger than the capacity of the single-insert specimen. This was not a surprising result because the failure mode of the specimens containing thin-slab inserts was concrete cone breakout. This interaction of failure cones would occur if there was insufficient spacing between these types of inserts.

In addition to the increase in load capacity, Specimen 2-RCD-T-1-6 also had different overall behavior than Specimen 2-RC-T-1-6 (Fig. 19). The photographs of the failed specimens shown in Fig. 20 help to explain this behavior difference. As shown in Fig. 20, there was extensive cracking of...
Fig. 21. Load-deflection curves for center-insert specimens showing the effect of axial compression. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

Fig. 22. Load-deflection curves for edge-insert specimens showing the effect of insert type. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

Specimen 2-RCD-T-1-6 immediately before failure. Alternatively, only a moderate crack pattern with almost no radial cracks developed in Specimen 2-RC-T-1-6 before failure.

The extensive damage that occurred in Specimen 2-RCD-T-1-6, compared with the damage in Specimen 2-RC-T-1-6, resulted in a drop in the load-carrying ability of the specimen after reaching the maximum load. Alternatively, Specimen 2-RC-T-1-6 exhibited stable behavior after reaching its maximum load, as this specimen developed only moderate damage while reaching the maximum load.

Influence of Axial Compression—Figure 21 shows the effect of increasing the axial compressive stress from 1 ksi to 2 ksi (6.9 MPa to 13.8 MPa) on the load-deflection behavior of the loop and thin-slab inserts. Doubling the amount of axial compression did not cause a significant change in the specimens’ load-deflection behaviors. The increase in load capacity of the specimens was 26% for those containing loop inserts and 11% for those with thin-slab inserts. Because the behaviors of specimens containing loop inserts were mostly governed by the deformation and failure of the inserts, the increase in friction between the insert and the surrounding concrete was the likely reason for the increase in specimen capacity with increasing axial compression.

The difference in failure mode may be why the increase in load capacity was less for the thin-slab insert (concrete failure) compared with that of the loop-insert specimen (steel failure). Following the formation of the concrete breakout cone in Specimens 2-RC-T-1-6 and 2-RC-T-2-6, the load in the specimen was carried through the inserts reacting against the longitudinal rods. In this load-carrying mechanism, any increase in friction between the inserts and the surrounding concrete (due to the increased axial compressive stress on the specimen) would not affect the specimen’s load capacity. In the case of the loop inserts, the increase in friction resulted in an increase in load capacity, as there was no concrete breakout in Specimen 2-RC-L-2-6 (Fig. 21).

Reinforced Edge-Insert Specimens

The effect of insert type on the behavior of specimens with limited edge distance is shown in Fig. 22. The figure...
shows the results for the four types of inserts tested with 1 ksi (6.9 MPa) axial compressive stress and with stirrups spaced at 6 in. (152 mm). The general behavior of the reinforced edge-insert specimens was similar to that of the corresponding reinforced center-insert specimens for each insert type.

The specimen with the loop insert (Specimen 2-RE-L-1-6) exhibited ductile behavior and was able to maintain near its maximum load for large values of insert displacement; however, it also had the lowest load capacity. The specimen with the thin-slab insert (Specimen 2-RE-T-1-6) also showed ductile behavior compared with the specimens containing double-leg and bolt inserts. The specimen with the bolt insert had the highest load capacity. The maximum load carried by this specimen was 78%, 42%, and 33% larger than the load carried by specimens with loop, thin-slab, and double-leg inserts, respectively.

Thin-Slab-Insert Specimens—Figure 23 presents the results from all of the thin-slab, edge-insert specimens tested. The figure shows the effects of axial compression, stirrup spacing, and the number of inserts. As shown, the specimens were tested under three levels of axial compression (2 ksi [13.8 MPa], 1 ksi [6.9 MPa], and no compression) and two stirrup spacings (6 in. and 4 in. [152 mm and 102 mm]).

As the load-deflection plots show, the specimen with no axial compression (Specimen 2-RE-T-0-4) and a 4 in. (102 mm) stirrup spacing not only had the smallest load capacity but also experienced a loss in capacity with increasing insert displacement. Specimen 2-RE-T-0-4 also exhibited the most concrete damage compared with the other single-edge-insert specimens.

For the thin-slab, edge-insert specimens, increasing the axial compressive stress from 1 ksi to 2 ksi (6.9 MPa to 13.8 MPa) while keeping the stirrup spacing at 6 in. (152 mm) (Specimen 2-RE-T-1-6 versus Specimen 2-RE-T-2-6) resulted in an approximate 11% increase in load capacity without changing the shape of the load-deflection curve significantly. Coincidentally, decreasing the stirrup spacing from 6 in. to 4 in. (152 mm to 102 mm) while keeping the axial compressive stress constant at 1 ksi (6.9 MPa) (Specimen 2-RE-T-1-6 versus Specimen 2-RE-T-1-4) also resulted in an approximate 11% increase in load capacity with no major change in the shape of the load-deflection curve.

Specimen 2-RED-T-1-6 had two thin-slab inserts with 3.75 in. and 7.75 in. (95 mm and 197 mm) edge distances. The load capacity of this specimen was 61% larger than that of Specimen 2-RE-T-1-6, which had only a single insert with a 3.75 in. (95 mm) edge distance. The fact that the addition of a second insert resulted only in a 61% increase in load capacity indicates the occurrence of interaction of the thin-slab inserts spaced at 4 in. (102 mm).

As shown in the graph in Fig. 23, the double-insert specimen suffered from a rapid loss of capacity after reaching the maximum load, unlike the single-insert specimens (2-RE-T-1-6, 2-RE-T-2-6, and 2-RE-T-1-4). The reason for the loss of capacity in the double-insert specimen was the extensive concrete damage that occurred while testing this specimen. Concrete breakout occurred over the entire face of the double-insert specimen, with the breakout region for one insert intersecting (or disturbing) that of the other insert. This mechanism resulted in deterioration of the load-carrying ability of the inserts after the specimen reached its maximum load.

Loop-Insert Specimens—Figure 24 shows the results from the loop, edge-specimens. The variables addressed in this figure are the same as those explained in Fig. 23. Similar to what was observed for the center-insert specimens, all of the edge-insert specimens with loop inserts exhibited ductile behavior.

Specimen 2-RE-L-0-4, which had stirrups spaced at 4 in. (102 mm) and was tested with no axial compression, exhibited a slightly different behavior than the other single-insert specimens. The failure pattern for Specimen 2-RE-L-0-4 was different from that of the other single-insert specimens. As seen, Specimen 2-RE-L-0-4 had a relatively deep breakout cone with large inclination angles.

There was a trend in the failure behavior of these specimens: As the axial compressive stress increased, the plan area of the concrete breakout region increased, while the angle of inclination decreased (that is, it flattened). Specimen 2-RE-L-0-4, which had stirrups spaced at 4 in. (102 mm) and was tested with no axial compression, exhibited a slightly different behavior than the other single-insert specimens. The failure pattern for Specimen 2-RE-L-0-4 was different from that of the other single-insert specimens. As seen, Specimen 2-RE-L-0-4 had a relatively deep breakout cone with large inclination angles.

LOAD-CARRYING BEHAVIOR OF SPECIMENS

Figures 25 through 28 give summaries of the specimens’ load capacities. In addition to the peak load values carried by the specimens, the figures also include loads carried when insert displacements were equal to 0.05 in., 0.1 in., and 0.2 in. (1.3 mm, 2.5 mm, and 5.1 mm). Even though these figures include the same information as the load-deflection plots shown previously, the bar charts provide an easy comparison of the load-carrying capacities of the specimens.

Fig. 24. Behavior of edge-insert specimens with loop insert. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 ksi = 6.89 MPa.
Figure 25, which includes the results from the unreinforced center-insert specimens, clearly shows the ability of loop inserts to maintain maximum load through large values of insert displacement, while the other three types of inserts suffered from rapid capacity loss. Similarly, Fig. 26 clearly reveals the large load capacities and relatively poor ductility of the bolt-insert specimens.

Figure 27 compares the load capacities for edge-insert specimens with four different types of inserts. As depicted in the figure, the specimen with the bolt insert had the largest load capacity, while the specimens with thin-slab and loop inserts were able to maintain almost full load capacity for insert deflection values of 0.2 in. (5.1 mm).

Figure 28 shows the effects of changing the axial compressive stress and stirrup spacing in edge-insert specimens with thin-slab and loop inserts. As depicted in the figure, increasing the axial compressive stress from 1 ksi to 2 ksi (6.9 MPa to 13.8 MPa) at a 6 in. (152 mm) stirrup spacing resulted in slight increases in load capacities in specimens with both thin-slab and loop inserts. Reducing the stirrup spacing from 6 in. to 4 in. (152 mm to 102 mm) with 1 ksi (6.9 MPa) axial compressive stress resulted in a similar effect.

As shown in Fig. 28, specimens tested without axial compression had the smallest capacity in the case of both thin-slab and loop inserts, but this effect was much less pronounced for the loop-insert specimens.

**PREDICTED LOAD CAPACITIES**

Load capacities of the unreinforced center-insert specimens were computed according to the procedures included in ACI 318-05, Appendix D, and the PCI Design Handbook and were compared with measured load capacities.28

**Background on ACI and PCI Procedures**

In ACI 318, the effective anchor embedment depth $h_{ef}$ is defined for anchors that can be considered wire formed (for example, L- and J-type anchors), while in provision RD.2.2 it is stated that "the wide variety of shapes and configurations of specialty inserts makes it difficult to prescribe generalized tests and design equations for many insert types. Hence, they have been excluded from the scope of Appendix D." Even though it is not clear whether the equations given in ACI 318-05 Appendix D are applicable to wire-formed inserts such as the ones tested in this study, the predicted load capacities are included for the sake of comparison.

In the procedure given in ACI 318, which is based on the concrete capacity design (CCD) method, the concrete breakout strength of a single anchor with an embedment less than 11 in. (280 mm) and located away from the edges in cracked concrete is calculated with Eq. (1), or Eq. (D-7) in ACI 318.

$$N_b = k_c \sqrt{f'_c h_{ef}^{1.5}}$$  \hspace{1cm} (1)

where

- $k_c$ = taken as 24 for cast-in anchors
- $f'_c$ = the design compressive strength of concrete
- $h_{ef}$ = the effective anchor depth

In the cases where no cracking is expected in the region where the anchor is located, the capacities obtained from Eq. (1) are multiplied by a factor ($\psi_{CN}$) that accounts for the absence of cracking. The factor $\psi_{CN}$ is specified as 1.25 for cast-in anchors.

The equation used in the PCI Design Handbook to calculate the concrete breakout strength of headed studs in tension is based on the work done by Shaikh and Yi.10 Shaikh and Yi indicated the applicability of this equation to different types of concrete inserts by stating that "Although the paper focuses on welded headed studs, the design equations would also apply to nut/washer anchor bolts and other similar inserts, such as loop inserts and expansion anchors."

Both the third and fourth editions of the PCI Design Handbook include sections on concrete breakout strength of wire-formed inserts,11,12 and it is recommended that the same equation that is given for the capacity of headed studs to predict the breakout capacity of wire-formed inserts be used. The fifth edition of the PCI Design Handbook, on the other hand, does not mention the capacity of wire-formed inserts, and it is not mentioned whether the concrete breakout capacity...
equation given for headed studs can be used for wire-formed inserts.

In the PCI Design Handbook method, concrete breakout strength of an anchor in tension is determined by multiplying the surface area of the concrete failure cone assuming a 45-degree inclination angle with an average uniform tensile stress of \( \frac{f_c}{\sqrt{3}} \) acting on the failure surface. Therefore, for a normal-weight concrete, Eq. (6.5.2) in the fifth edition of the PCI Design Handbook is:

\[
P_c = A_0 \left( 2.8 \lambda \sqrt{f_c} \right)
\]

where

- \( A_0 \) = the surface area of the 45-degree failure cone
- \( \lambda = 1.0 \) for normal weight concrete

For a single anchor away from free edges, the failure surface area is:

\[
A_0 = \sqrt{2} \cdot l_e \pi \left( \frac{f_c}{\sqrt{3}} + d_h \right)
\]

where

- \( l_e \) = the embedment depth of the anchor
- \( d_h \) = head diameter of the anchor

Table 4 gives the geometric properties of the inserts and the measured concrete strengths used to predict the breakout capacities. The embedment depth and head diameter of the inserts used in Eq. (3) were determined based on Fig. 6.5.8 of the fourth edition of the PCI Design Handbook.\(^{12}\) For loop, thin-slab, and double-leg inserts, the effective depth values used in the calculations were the total embedment depth of the inserts minus the diameter of the wires. For bolt inserts, the effective depth was taken as the depth of the insert to the top of the washer.

Even though ACI 318-05 Appendix D allows using a larger failure area to compute the concrete breakout capacity when there is a washer or plate added at the head of the anchor, the capacity of bolt inserts was determined with Eq. (1) without considering this increase in failure area (because the thickness of the washers added at the head of the bolts was small). The effect of the washer was also ignored when computing the capacity according to the PCI Design Handbook, and the head diameter of the bolt was used rather than the diameter of the washer.

**Comparison of Measured and Predicted Capacities**

Table 4 presents the concrete breakout strengths of the unreinforced center-insert specimens calculated using Eq. (1) and (2). Figure 29 shows the bar-chart representation of the comparison between the measured and calculated capacities.

It should be noted that the measured load capacity for Specimen 1-UC-L-0-N was not the concrete breakout capacity, as this specimen experienced steel failure. As a result, the calculated and measured load capacities given in Table 4 for this specimen should not be compared.

As shown in Fig. 29, the PCI Design Handbook equation, or Eq. (2), overestimated the concrete breakout capacity of the specimens, with the level of overestimation being higher for the bolt-insert specimens. The average value of the ratio of predicted to measured capacities was 1.25 with a coefficient of variation of 14%.

Good agreement was observed between the measured concrete breakout capacities and those calculated using Eq. (1) considering the case of cracked concrete. When the \( \psi_{CN} \) factor was applied to account for the uncracked concrete case, the calculated capacities overestimated the measured values. For the case with no \( \psi_{CN} \) factor (that is, assuming cracked concrete), the ratio of predicted-to-measured breakout capacities had an average value of 1.02 with a coefficient of variation of 13%. When the \( \psi_{CN} \) factor was included in the calculations, the average value of the ratio of predicted-to-measured breakout capacities became 1.27 with a coefficient of variation of 13%, similar to the results in the PCI Design Handbook.

The reason that Eq. (1) with no \( \psi_{CN} \) factor predicted the concrete breakout strength of the test specimens with accuracy may be the test setup used in the study. As mentioned, the specimens were supported at the top and bottom of the back side by threaded rods embedded in the concrete blocks. In this type of setup, bending of the specimen, which creates tensile stresses at the insert location, occurs as the ten-
Table 4. Comparison of Measured and Predicted Concrete Breakout Capacities for Unreinforced Specimens

<table>
<thead>
<tr>
<th>Specimen Designation</th>
<th>$h_o$, in.</th>
<th>$d_o$, in.</th>
<th>Concrete Strength, psi</th>
<th>Measured Capacity, kip</th>
<th>Capacity Predicted by Eq. (1), kip</th>
<th>Capacity Predicted by Eq. (2), kip</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Cracked</td>
<td>Uncracked</td>
</tr>
<tr>
<td>1-UC-T-0-N</td>
<td>3.3</td>
<td>1</td>
<td>10,600</td>
<td>15.3</td>
<td>14.8</td>
<td>18.5</td>
</tr>
<tr>
<td>1-UC-L-0-N*</td>
<td>3.8</td>
<td>0</td>
<td>10,600</td>
<td>14.9</td>
<td>18.3</td>
<td>22.9</td>
</tr>
<tr>
<td>1-UC-D-0-N</td>
<td>3.1</td>
<td>1.125</td>
<td>10,600</td>
<td>17.0</td>
<td>13.5</td>
<td>16.9</td>
</tr>
<tr>
<td>1-UC-B-0-N</td>
<td>3.7</td>
<td>1.25</td>
<td>10,600</td>
<td>17.1</td>
<td>17.6</td>
<td>22.0</td>
</tr>
<tr>
<td>2-UC-T-0-N</td>
<td>3.3</td>
<td>1</td>
<td>8700</td>
<td>13.6</td>
<td>13.4</td>
<td>16.8</td>
</tr>
<tr>
<td>2-UC-L-0-N</td>
<td>3.8</td>
<td>0</td>
<td>8850</td>
<td>14.2</td>
<td>16.7</td>
<td>20.9</td>
</tr>
<tr>
<td>2-UC-D-0-N</td>
<td>3.1</td>
<td>1.125</td>
<td>9000</td>
<td>12.2</td>
<td>12.4</td>
<td>15.5</td>
</tr>
<tr>
<td>2-UC-B-0-N</td>
<td>3.7</td>
<td>1.25</td>
<td>9150</td>
<td>14.1</td>
<td>16.3</td>
<td>20.4</td>
</tr>
</tbody>
</table>

*Steel failure. (Breakout strength is not applicable.) Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

Concrete load is applied to the insert. Even though the specimens were uncracked prior to the load tests, existence of tensile stresses at the insert location due to bending of the specimens is probably the reason there is better agreement between the measured capacities and those predicted assuming cracked concrete rather than uncracked concrete.

Concrete breakout capacities were also calculated for the reinforced specimens tested under 1 ksi (6.9 MPa) of compressive stress. In those cases, the predicted capacities using Eq. (1) with the $\psi_{CN}$ factor of 1.25 underestimated the measured capacities. This was expected due to the beneficial effects of the reinforcement and the prestress, which promotes resistance to cracking.

Observations on the Behavior of Loop-Insert Specimens

A tension test was performed on a piece of wire cut from a loop insert in order to measure the steel strength of these inserts. Strength of the wire was measured to be 8.4 kip (37 kN), which means that without any effect of additional specimen reinforcement and axial compression, the strength of a loop insert in the case of steel failure mode is expected to reach up to 16.8 kip (74.7 kN), assuming that both legs of the loop inserts fail simultaneously. As it is unlikely that both legs of the insert would fracture simultaneously, the capacity of the inserts would be expected to range from 8.4 kip to 16.8 kip (37 and 74.7 kN) due to load sharing. Moreover, during load testing of Specimen 1-UC-L-0-N, fracture of the insert legs at corner locations occurred, which indicates the involvement of bending as well as axial tension in the failure process. As a result, the strength of a loop insert itself is expected to be less than that corresponding to the pure tensile strength of the insert legs. The measured strength of 14.9 kip (66.3 kN) for Specimen 1-UC-L-0-N was consistent with these expectations.

The load-carrying mechanism of loop inserts explained previously assumes that there was no concrete contribution to the steel failure capacity. However, because fracture of the insert legs occurred inside the concrete, some part of the maximum load reached by the loop-insert specimens was due to concrete contribution, even though the final failure of the specimen was due to fracture of the insert. For example, Specimen 1-RC-L-1-6 shown in Fig. 17 reached a maximum load of 17.9 kip (79.4 kN) during load testing. At this point, breakout of cover concrete occurred with some level of cracking. Following the breakout of cover concrete, the load carried by the specimen dropped and became stable at approximately 13.5 kip (60.0 kN). The breakout of cover concrete caused the part of the load carried by concrete to be transferred to the
insert. Therefore, it may be said that the difference between the maximum load and the value of load attained after concrete breakout corresponded to the contribution of concrete to the load capacity of Specimen 1-RC-L-1-6.

In light of the previous discussion on the load-carrying mechanism, it can be said that in the case of steel failure mode (fracture of insert legs), the load capacity of loop inserts might be higher than the total tensile strength of the insert legs in the presence of reinforcement and axial compression. The presence of reinforcement and axial compression would increase the resistance against cracking of concrete and delay the initiation of cracking around the insert location. The delayed initiation of cracking and, hence, breakout of cover concrete would result in an increase in the maximum load, even though the load at which the insert fracture occurs might remain unchanged. As a matter of fact, an analysis of the test results given in Table 2 indicates that for specimens with a loop insert, the load capacity of specimens increased slightly with increasing axial compressive stress and decreasing stirrup spacing, even for the specimens that had steel and weld failures.

Among the loop-insert specimens that had steel failure, the load capacities of Specimens 1-RC-L-1-6 and 2-RC-L-2-6 corresponded to 107% of the total tensile strength of the insert legs (16.8 kip [74.7 kN]). This observation supports the theory explained previously that the load capacity of a loop insert might be higher than the total tensile strength of the insert legs in the presence of reinforcement and axial compression. For Specimen 1-UC-L-0-N, which had no reinforcement and was tested with no axial compression, the ratio of measured capacity to the total tensile strength of the insert legs remained at 89%.

**SUMMARY AND CONCLUSIONS**

Concrete, steel, and weld failures were observed during load testing of the insert specimens. Concrete failures occurred in all specimens with thin-slab, double-leg, and bolt inserts. Considerable plastic deformation, including necking at several locations, was observed to occur in specimens with loop inserts. The majority of the specimens with the loop insert (7 specimens out of 11) failed by fracture of steel wires and/or welds.

Specimens with loop inserts exhibited stable behavior with almost no degradation of load-carrying capacity after reaching their peak loads. As the behavior of these specimens was governed by the deformation (and eventual fracture, in the majority of cases) of the inserts, the presence of longitudinal reinforcement and axial compression resulted in slight increases in load capacities without changing the global behavior of the specimens.

The test results also indicate there is no significant effect of limited edge distance on specimens with loop inserts. The capacity of single-loop-insert specimens doubled with the addition of a second insert for both the center-insert and edge-insert cases. It was concluded that loop inserts were the most reliable and robust of the four types tested; however, they also demonstrated the lowest tensile force capacities for reinforced specimens.

The load capacities of the loop-insert specimens that exhibited steel failure were generally observed to be smaller than the total tensile strength of the legs of the loop insert. The two reasons for this reduction in load capacity were that both legs of the loop insert did not fracture simultaneously and bending, in addition to a pure tension load, was involved in the fracture of legs. Alternatively, the beneficial effects of the presence of reinforcement and axial compression resulted in some of the loop-insert specimens exhibiting steel failures after achieving loads as much as 7% higher than the total tensile strength of the insert legs.

The behavior of the specimens with thin-slab, double-leg, and bolt inserts was influenced by the presence of reinforcement. Unreinforced specimens exhibited brittle behavior with limited deformation taking place before failure. The addition of reinforcement resulted in some level of improved ductility based on the type of insert.

Significant increases in capacity were noted in reinforced bolt-insert specimens compared with the unreinforced specimens. Specimens containing thin-slab, double-leg, and bolt inserts exhibited slight increases in strength with increasing amounts of axial compression. Reducing the stirrup spacing resulted in similar effects to increasing the axial compression. Interaction of failure surfaces was also determined to occur in double-insert specimens.

Failure cone inclination angles observed in the unreinforced specimens ranged from 29 degrees to 45 degrees and were consistent with the inclination angles assumed by several sources to predict the concrete breakout strength of cast-in-place headed anchors.

The measured concrete breakout capacities were found to be in good agreement with the capacities predicted by Eq. (1), Eq. (D-7) of ACI 318-05 Appendix D, when the factor to account for the absence of concrete cracking was not used. The measured capacities agreed more with the breakout capacity in cracked concrete than the capacity in uncracked concrete due to tensile stresses created on the front face of specimens because of possible bending of the specimens during testing.

**RECOMMENDATIONS**

Based on this study, the following recommendations are offered:

- If the maximum tensile force requirements for an insert are known during design, it is possible to select a bolt insert configuration, or possibly thin-slab or double-leg insert configurations, to meet the tensile requirements. The bolt insert, given its high tensile force capacity in reinforced concrete, also offers the option to select connector strength (steel threaded rod or reinforcing bar that is connected to the insert) such that the connector yields before the bolt insert would exhaust its capacity. However, maximum loads for inserts are seldom known with accuracy, in which case the loop insert may offer a better choice given its ductile behavior and insensitivity to edge distance, presence and detailing of

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reinforcement, and amount of axial compression.

- In the case of steel failure of loop inserts, the load capacity of the insert might be estimated to be 80% to 90% of the total tensile strength of the insert legs. In the presence of reinforcement and axial compression, the insert capacity might be increased to 100% to 110% of the total tensile strength of the insert legs, or for more conservative estimates, the beneficial effects of reinforcement and axial compression might be ignored.

- *PCI Design Handbook* 5th Edition Eq. (6.5.2) should not be used to predict the concrete breakout capacity of the types of inserts tested in this study because this equation resulted in estimates of the measured insert capacities that were not conservative.

- Equation D-7 of ACI 318-05 Appendix D may be used to predict the concrete breakout capacity of the types of inserts tested in this study. However, care must be exercised in using the \( \psi_{CN} \) factor to account for the absence of cracking in the concrete. Even though the inserts may be placed in uncracked regions, the concrete breakout capacity of inserts may be better predicted by the breakout capacity in cracked concrete if tensile stresses are likely to develop during loading at the insert locations. It should be assumed that tensile stresses are likely to develop at insert locations if there is not a reliable and sustained compression field present in the concrete, such as that from prestress.

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